HYDRAULICS CHAPTER 300

SECTION 301 INTRODUCTION

301.01 Section Description

This chapter provides policies and technical procedures for analyzing the majority of stormwater facilities required for land alteration projects. However, more detailed analyses may be required depending on the specific site characteristics.

301.02 Definitions

Culvert: A straight length of conduit which conveys stormwater or stream flow through a roadway embankment or past some other type of flow obstruction.

Critical Depth: Critical depth is the depth of flow at which the specific energy is a minimum. An illustration of critical depth is the depth at which water flows over a weir when no other backwater forces are involved. For a given discharge and prismatic cross-section geometry there is only one critical depth.

Free Outlets: Free outlets are those outlets whose tailwater is equal to or lower than critical depth. For culverts and storm drains having free outlets, lowering of the tailwater has no effect on the discharge or the backwater profile upstream of the tailwater.

Freeboard: Freeboard is an additional depth regarded as a safety factor, above the peak design water elevation.

Invert: Invert refers to the flowline of the culvert (inside bottom).

Specific Energy: Specific energy (sometimes called "specific head") is defined as the sum of the depth and velocity head of the flow.

Steep and Mild Slope: A steep slope culvert operation is where the computed critical depth is greater than the computed uniform depth. A mild slope culvert operation is where critical depth is less than uniform depth.

Storm Drains: Underground pipe systems designed to intercept and convey to an adequate outlet stormwater runoff.

Submerged: Submerged inlets are those inlets having a headwater greater than (1.5D).

Submerged Outlets: Partially submerged outlets are those outlets whose tailwater is higher than critical depth and lower than the height of the culvert. Submerged outlets are those outlets having a tailwater elevation higher than the crown of the culvert.

Tailwater: Standing or running water, and specifically its elevation, outside the downstream or outlet end of a culvert or storm drain system.

Uniform Flow: Uniform flow is flow in a prismatic channel of constant cross section having a constant discharge, velocity and depth of flow throughout the reach. In uniform flow it is assumed that the depth of flow is the same at every section of the channel.

To provide consistency within this chapter the following symbols will be used. These symbols were selected because of their wide use in hydrologic and hydraulic publications. In some cases the same symbol is used in existing publications for more than one definition. Where this occurs in this chapter, the symbol will be defined where it occurs in the text or equations.

TABLE 301-1: Symbols and Definitions

Symbols	Definition	Units
A	Cross section area	ft^2
C	Rational method	-
C_{SF}	Stability correction factor	-
C_{SG}	Specific gravity correction factor	-
C_D	Weir coefficient or discharge coefficient	-
D	Depth of flow	ft
D_i	Size of riprap for which i (e.g. 100, 50, or 15) percent of the stone diameters are smaller	ft
g	Acceleration due to gravity	ft/s^2
H	Head on orifice	ft
H_L	Sum of energy losses	-
HW	Headwater depth above invert of culvert (depth from inlet invert to upstream total energy grade line)	ft
i	Rainfall intensity	in/hr
K	Side slope correction factor	-
L	Length	ft
n	Manning roughness coefficient	-
P	100-year rainfall	in
P_w	Wetted perimeter	ft
Q	Rate of runoff/discharge	cfs
Q_r	Allowable release rate	cfs
R or r	Hydraulic radius	ft
S or s	Bed Slope or Slope of hydraulic grade line	ft/ft
S_f	Friction slope	ft/ft
S_G	Specific gravity of stone	lbs/ft^3
S_x	Cross slope	ft/ft
SF	Stability factor	-
t	Storm duration	min
t_c	Time of concentration	min

T_d	Shear stress	lbs/ft^3
T_p	Permissible shear stress	lbs/ft^3
T	Channel top width	ft
T_w	Tailwater depth	ft
v	Velocity	ft/s
Vol, or V	Volume	ft^3
V_f	Huff storm factor	

SECTION 302 DETENTION/RETENTION DESIGN

302.01 Introduction

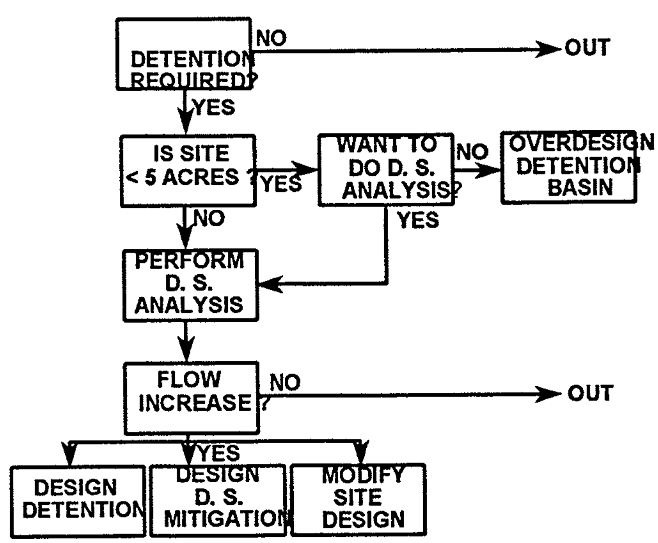
The design methods and criteria outlined within this section shall be used in the design and evaluation of detention/retention systems within the jurisdictional boundaries of this Manual. All designs must be supplemented with a detention/retention design summary report. A sample detention/retention design summary report is provided by Figure 302-1.

302.02 The requirement for Detention/Retention

Detention/retention shall be required on all new developments except:

1. those identified in Chapter 200, section 201.06; or
2. for the cases where downstream analysis indicates none is required; or
3. where downstream mitigation efforts are accepted in lieu of detention/retention.

The figure below shows the logical steps in considering the need for detention/retention:



STEP 1: Is detention/retention required according to 201.06? If it is not the designer skips any consideration of downstream analysis or detention/retention. If downstream analysis is required the designer

proceeds to step 2. The downstream analysis must assess <u>each</u> outflow point from the site separately.

STEP 2: Are any individual potential detention/retention sites draining areas less than 5 acres total including off-site drainage? In this step the designer begins to assess the site in terms of regional or local detention/retention basin(s). If a particular outflow point's drainage area is less than 5 acres the designer has the option of avoiding downstream analysis in return for detention/retention overdesign for that part of the site.

STEP 3: If the drainage area is less than 5 acres the designer has an option. Does the designer want to perform downstream analysis? There may be good reasons to avoid downstream analysis in favor of increased detention design. If "NO" the designer performs an oversized detention/retention design and proceeds with the design. If "YES" the designer goes to step 4.

STEP 4: If the drainage area is equal to or greater than 5 acres the designer performs a downstream analysis for the storm frequencies required for the stormwater facilities encountered downstream to the ten percent point. This will normally require the 10-, 25- and 100-year storms.

STEP 5: If the downstream system is adequate according to 201.07 the designer needs to provide no detention/retention, mitigation or site modification. If not the designer proceeds to step 6.

STEP 6: If the site is not adequate the designer performs one or a combination of detention/retention design, downstream stormwater facility mitigation and/or site modification.

302.03
Minimum Performance
Level of Detention/
Retention Facilities

The design criteria for detention/retention is:

1. **<u>Local basins</u> are those which have a total land area contributing flow to the detention/retention basin, including on-site and off-site areas, of less than five (5) acres. Minimum hydraulic performance levels and accepted design methodologies for local basins shall conform to the following:**

* **release, at a minimum, the peak discharge resulting from the 100-year design storm event runoff at the 10-year design storm event runoff peak discharge rate for existing watershed conditions.**

* **release the peak discharge resulting from a 2-year design storm event runoff from the contributing watershed area at the 2-year design storm event runoff peak discharge rate for existing watershed conditions.**

Local basin designs where the designer elects to overdesign the detention basin in lieu of performing downstream analysis, may be designed using the Modified Rational method as set forth herein. All other detention/retention designs shall use runoff hydrographs and routing techniques.

2. Regional basins are those which have a total land area contributing flow to the basin, including on-site and off-site areas, of five (5) acres or larger. Minimum hydraulic

performance levels and accepted design methodologies for regional basins shall conform to the following:

* release runoff resulting from a 2-year design storm event over the entire contributing watershed for post-development conditions at a peak rate and velocity no greater than the peak rate and velocity from the 2-year design storm event runoff based on the pre-development watershed conditions.

* the developed site runoff during the runoff from both the 10-year and 100-year design storm events shall be designed to be released at a peak rate and velocity no greater than the peak rate and velocity from the 10-year design storm event runoff based on pre-development watershed conditions.

All detention/retention designs shall use runoff hydrographs and routing techniques.

302.04
Increased Detention/
Retention In Lieu Of
Downstream Analysis

Performing downstream analysis creates an opportunity to avoid the need for detention/retention altogether or reduce the detention/retention requirement in favor of downstream mitigation. However, an increased level of detention/retention may be used in lieu of the downstream analysis described in Chapter 200, sections 201.05 through 201.08. The design shall conform to the requirements for a local basin plus:

* The developed stormwater runoff for the 2-year storm event and all upstream contributing land areas under existing conditions shall be detained to the 50 percent of the existing 2-Year discharge from the developing site.

* All impervious area on the site shall be routed through the detention/retention facility.

* Stormwater flows shall not be diverted to downstream facilities which do not accept runoff from the developing property under existing conditions.

* Minor collector swales located within residential or commercial developments, or collector swales located within open land uses such as agricultural fields, golf courses, and parks and recreation areas, as examples, will not be considered acceptable outfalls for a detention/retention providing this level of runoff control, unless a low flow system is installed downstream to convey trickle flows from these basins.

The Modified Rational method, discussed in the next section, may be used for sizing this type of detention pond since downstream analysis is not required. Standard routing design may also be used even though downstream analysis is not done.

302.05 Modified Rational Method

The Modified Rational method may be used for the sizing of local detention/retention basins where downstream analysis is not required, as described in this section. The method approximates the required storage volume of the detention basin during the storm event. In addition the hydrograph shape is trapezoidal to approximate the shape of other hydrologic methods. Figure 302-2 illustrates the hydrograph and basin outflow. The general equation for the storage volume for this method is:

$$Vol=60[CiAt-Q_R(t+t_c)/2] \qquad \text{(Equation 302.01)}$$

where:

Vol = required volume of the pond (cubic feet)
C = post development C factor
i = rainfall intensity from the IDF curve (in/hr)
A = Area
Q_R = allowable release rate
t = storm duration to maximize the storage volume (minutes)
t_c = post-developed time of concentration (minutes)

To properly design a retention/detention basin system two critical design criteria must be applied. These two design criteria are the 100-year post developed peak flow controlled to the 10-year pre-developed peak flow levels and the 2-year post-developed peak flow controlled to 50 percent of the 2-year pre-developed peak flow levels.

It should be noted, that for this method to give accurate results comparable to those found for routing of hydrographs, accurate estimates of the pre-development peak flows using the Rational Method must be made. The runoff coefficients given in Table 204.01 are conservative for peak flow design in that they are at the high end of the range of values possible for undeveloped conditions. While this provides a measure of conservatism for peak flow conveyance structures, it is very nonconservative for detention design where values at the low end of the range are more applicable. Therefore the following C values (called C_R) are to be used to determine the pre-development allowable release rates (Q_R) for detention design for the Modified Rational method. These values are to be used only for pre-development conditions for the Modified

Rational method detention design, not for peak flow calculations for stormwater conveyance facilities such as storm drains or culverts.

C_R Factors for Allowable Release Rates
for Modified Rational Detention Design

	Hydrologic Soil Type B		Hydrologic Soil Type C	
Slope (%) →	0-2	2-7	0-2	2-7
2 - year	0.03	0.03	0.03	0.03
10 - year	0.03	0.04	0.11	0.13

Thus, for the 10-year pre-development allowable release rate the equation for Q_R is $C_R\, i_{10}\, A$. And for the 2-year pre-development allowable release rate the equation for Q_R is $0.5\, C_R\, i_2\, A$ (allowing for fifty percent of the 2-year flow).

Equation 302.01 can be combined with a form of the IDF curve equations, derivatives taken and set to zero and resubstitution done. The result is two equations that can be used to size a detention/retention basin for both design criteria. Contact the Department for more information on the method's derivation. To control the 100-year post-development peak flow to the 10-year pre-developed peak flow Equation 302.02 should be used to determine the critical duration, the time at which the storage volume is maximized.

$$t=\sqrt{\frac{2CAab}{Q_R}}-b \qquad \text{(Equation 302.02)}$$

where:

C = developed condition C factor
t = critical storm duration (minutes)
a = constant for 100-year storm event in Indianapolis = 222.37
b = constant for 100-year storm event in Indianapolis = 18.48
Q_R = allowable release rate, cfs
A = area in acres

The maximum required volume for the 100-year post-developed to 10-year pre-developed design can be found using Equation 302.03.

$$V_{max}=60[CAa-(2CabAQ_R)^{1/2}+\frac{Q_R}{2}(b-t_c)] \qquad \text{(Equation 302.03)}$$

where: V_{max} = required storage volume (cubic feet)

and all other variables are as previously defined.

In a similar fashion, the preceding formulas should be used to size a 2-year storm event orifice and storage volume. These 2-year values (storage volume and allowable peak outflow) can be used to compute a smaller 2-year orifice with an overtopping weir wall placed in front of the 10-year orifice. The designer should exercise care to ensure that the weir wall is not more restrictive than the 10-year orifice. The critical duration for this design criteria, the time at which the storage volume is maximized, can be found solving Equation 302.02 with the following variable definitions:

a = constant for 2-year storm event in Indianapolis = 91.28
b = constant for 2-year storm event in Indianapolis = 14.92

The maximum required volume for the 2-year post-developed peak flow to fifty percent of the 2-year pre-developed peak flow design can be found using Equation 302.03 with the appropriate 2-year design constants and release rate.

The results of these prior computations are two sets of data. The first is the required storage volume and associated allowable outflow for the 100-year post-developed peak flow release to the 10-year pre-developed peak flow design. The second is the required storage volume and associated allowable outflow for the 2-year post-developed peak flow reduce to fifty percent of the 2-year pre-developed peak flow design. Using this information, the designer can derive an outlet structure configuration that meets the allowable outflow criteria based on a maximum headwater depth that corresponds to the maximum required storage volume. Typical designs place the smaller 2-year orifice in front of the 10-year orifice with a weir wall set at the maximum 2-year headwater depth that corresponds to the 2-year volume.

After determining the headwater and orifice size, the required storage volumes calculated from equation 302-03 for both design criteria must be multiplied by a Huff Storm factor to account for greater volumes of rainfall for longer duration storms using Huff distributions. The factor is defined by Equation 302.04.

$$V_f = \frac{P_{8\text{-}hour}}{P_{Tcrit}} \qquad \text{(Equation 302.04)}$$

where:

V_f = Huff storm factor
$_{P\text{-}hour}$ = 100-year, 8-hour storm rainfall depth (in.)
= 4.77 in.
P_{tcrit} = 100-year rainfall depth for the critical storm duration as solved by equation 302.02

302.06
Bypassing Flow

When stormwater detention/retention is required, all parts of the developing site should drain through the detention/retention basin, unless otherwise approved.

302.07
Detention/Retention

The minimum accepted bottom transverse slope of dry detention

basins shall be 1.0 percent (1%). Vegetated bank side-slope shall be no steeper than 3 (horizontal) to 1 (vertical).

Vegetated areas of wet detention basins shall have an earthen embankment constructed with side slopes no steeper than 3 (horizontal) to 1 (vertical). Earthen embankments armored with rock rip-rap shall have side slopes no steeper than 1 ½ (horizontal) to 1 (vertical).

The maximum ponding depth for parking lot detention shall be Ten (10) inches for the 100-year storm event runoff from the entire contributing watershed.

Minimum normal depth of a wet pond, calculated as the deepest point in the pond, shall be eight (8) feet.

302.08
Design of D/R Facility
Emergency Spillway's

Emergency spillways shall be capable of handling one and one-quarter times the peak discharge and peak flow velocity resulting from the 100-year design storm event runoff from the entire contributing watershed, assuming post-development conditions, draining to D/R facility. However, engineering judgement may dictate use of a higher design standard. Many types of emergency spillway are allowable provided adequate provision is made for the discharge of the flow through the facility and a minimum freeboard of one-foot (1) is provided for larger regional ponds above the maximum anticipated flow depth through the emergency spillway.

302.09
Outlet Hydraulics -
Orifice Flow

The outlet hydraulics of a detention/retention basin typically consist of two types of flow, orifice and weir flow. The basic equation for determination of orifice flow is as follows:

$$Q=C_D A\sqrt{2gH} \qquad \text{(Equation 302.05)}$$

where:

- Q = peak discharge rate, cfs
- C_D = coefficient of discharge, dimensionless
- A = cross sectional area of orifice, square feet
- g = acceleration due to gravity (32.2 ft/sec/sec)
- H = head on the orifice, feet.

The value of H is determined by different methods depending upon the location of the water surface as follows:

Free Discharge: H is the difference in elevation between upstream water surface and center of flow of the orifice.

Submerged Orifice: H is the difference in elevation between upstream and downstream water surfaces.

The value of the coefficient of discharge C_D is a function of the size and shape of the orifice, the head on the orifice, the sharpness of the orifice's edge, the roughness of the inner surface, and the degree to which the contraction of flow is suppressed (Reference King's Handbook of Hydraulics). A nominal value of 0.60 may be used for the standard types of orifices and head ranges used for outlet control structures, however, sound engineering judgement must be used in the practical application of this value.

302.10
Outlet Hydraulics - Weir Flow

Weir structures may be either sharp-crested, rounded, or broad-crested. The means by which a weir functions can change depending upon the depth of head above the weir. A broad-crested weir may become a sharp-crested weir at higher heads, depending upon its physical configuration.

The general equation for weir flow is as follows:

(Equation 302.06)

$$Q=C_D L H^{1.5}$$

where:

Q = peak discharge rate, cfs
C_D = coefficient of discharge, dimensionless
L = length of the weir, feet
H = head on the weir, the difference in elevation between the weir crest and the water surface measured upstream of the crest a short distance, feet.

Values of C_D for sharp-crested, rectangular weirs can range from about 3.3 to 4.9. This coefficient is dependent upon the head on the weir, the height of the weir crest above the streambed, and the degree of submergence. Values of C_D can be selected from tables in King's Handbook of Hydraulics or other suitable references. Sound engineering judgement must be used in the interpretation of C_D values for various design conditions.

302.11
Easements

Guidelines for minimum easement widths are provided below. More stringent requirements for stormwater easement size and additional covenants may be made by the Department based upon individual site conditions.

Detention/retention basins which provide a "regional" benefit shall be constructed within a stormwater easement either platted or legally described and recorded as a perpetual stormwater

easement a minimum of twenty (20) feet horizontally outside of the design 100-year flood water elevation of the basin.

Detention/retention basins which do not provide a "regional" flood control benefit will not be required to be constructed within a stormwater easement.

Public street rights-of-ways will not be acceptable areas for construction of detention/retention facilities, unless otherwise approved by this Department.

SECTION 303 OPEN CHANNEL DESIGN

303.01
Introduction

Open channel flow may be evaluated utilizing Manning's equation, however, restrictions within open channels, such as at open culverts or storm drains, may be required to be evaluated by more sophisticated design methods such as the direct-step backwater or reservoir routing techniques.

303.02
Easements/Minimum Flood Protection Elevations

Hydraulic and hydrologic computations must be performed to determine the maximum inundated area resulting from the 25-year design storm event runoff. No habitable structures may be located within this area.

For areas which drain more than 5 acres:

* **easements must be dedicated which encompass the entire delineated 25-year flood area;**
* **additional hydraulic and hydrologic calculations must be performed to determine the maximum inundated area resulting from the 100-year design storm event runoff**
* **a 100-year flood line must be delineated in addition to the 25-year easement restriction.**

The lowest location of any proposed habitable structures where water may enter must be located above this delineated 100-year flood elevation.

Collector Swales: Surface water collector swales within the rear yard and side yard areas of residential subdivisions and on all non-residential parcels draining more than five (5) acres shall be constructed within a drainage easement possessing a minimum width of twenty (20) feet. For residential properties the drainage swale should be generally constructed approximately in the middle of the easement.

Open Ditches: Open ditches, those which do not have grass bottoms or are not accessible to vehicular traffic within the ditch, shall be placed within a drainage easement of a minimum width of ten (10) feet from the top of one bank of the channel.

Flood Protection Grades: Properties located within the regulatory floodway or floodway fringe area shall provide floodway/floodway fringe boundary delineations on the site plan. A citation of the regulatory source for these boundary delineations and minimum lowest enclosed floor elevations of permanent structures shall be provided on the site plan. Additional requirements for completing alterations to the land or existing structures within regulatory flood hazard areas may be found within the Flood Control District Zoning Ordinance for Marion County, Indiana, which may be found within the Appendix of Section 100 of this Manual.

303.03
Grading and Depth of Open Channels

Except for road side ditches, the side-slope of grass lined channels shall be no steeper than 3 (horizontal) to 1 (vertical). When the bottom width of trapezoidal grass-lined channels exceeds fifteen (15) feet, rock rip-rap or paved low flow channels shall be provided to convey low flows and to prevent meandering. For grass-lined channels, intended to convey continuous trickle flows such as for retention pond outlets, an enclosed storm drain, subsurface tile with gravel envelope, rock rip-rap, or paved low flow channel will be required.

The side-slope of rock riprap lined open conveyance channels shall be no steeper than 1 ½ (horizontal) to 1 (vertical), unless otherwise approved.

Concrete-lined channels shall be required by the Department as deemed necessary to either control erosion and/or eliminate wetness within open stormwater conveyance channels.

To prevent chronic wetness in the invert of open channels, subsurface tiles shall be installed a minimum of 1 ½ feet in depth (from the tile invert), with a #8 gravel or equivalent size washed stone as a granular envelope, as follows:

Minor drainage collector swales in rear yards and between homes shall possess a maximum channel length of 400 lineal feet, unless subsurface tile or swale invert treatment in the form of concrete paving is also provided. The required channel slope and invert treatment for minor drainage collector swales shall be as follows: grass lined swale if slope is 1% or greater and length is less than

400 feet; concrete paved channel or other acceptable treatment (such as a 2 ft. by 2 ft. trench filled with No. 8 washed gravel wrapped in fabric) if channel slope is between 0.3% and 0.5%, and/or length is greater than 400 feet; subsurface drainage tile if channel slope is between 0.5% and 1.0%, and/or length is greater than 400 feet. The minimum channel slope shall be 0.3%.

For relatively large open channels and perennial streams, minimum channel slopes and the provision of subsurface drainage shall be approved on a case basis by the Department.

Privately owned open channels, including man-made ditches, swales, and natural streams, shall be repaired and/or reconstructed such that all woody vegetation has been cleared, and the channel banks are properly stabilized to prevent present and future erosion.

303.04 Channel Lining Design Requirements

The peak discharge from the 10-year design storm event shall be used to design channel linings for all channels. The final design of open channels should be consistent with permissible shear stress (τ_p) for the selected channel lining. Reference should be made to the publication FHWA-RD-89-110, HEC-15 for a more detailed description of this analysis. Permissible shear stress for various channel linings are reported in Table 303-1, Figure 303-1 and Figure 303-2.

The process of channel lining design is as follows:

* Select a lining and determine the permissible shear stress (τ_p), in lbs/ft^2, from Table 303-1, Figure 303-1 and 303-2.

* Choose an initial Mannings "n" value based on engineering reference books, such as "Open-Channel Hydraulics" by V.T. Chow.

* Calculate normal flow depth (D), in ft, at design discharge using Mannings Formula.

* Compute maximum shear stress (T_d), in lbs/ft^2, at normal depth as:

$$T_d = 62.4DS \qquad \text{(Equation 303.01)}$$

where:

T_d = maximum shear stress (lbs/ft^3)
d = normal flow depth (ft)
S = channel gradient (ft/ft)

* If $T_d < T_p$, then the channel lining is acceptable. Otherwise consider the following options.
  * choose a more resistant lining
  * Use concrete, gabions, or other more rigid lining either as

full lining or composite

* decrease channel slope
* decrease slope in combination with drop structures
* increase channel width and/or flatten side slopes

For channel designs incorporating a riprap lining, the following procedures shall be used. Riprap shall not be placed on a side slope steeper than 1.5H:1V unless otherwise approved. The toe of the riprap shall be extended below the channel or ditch bed a minimum distance of one (1.0) foot or 1.5 D_{50} (which ever is greater) except where alternate methods are approved or where the ditch or channel bottom is also covered with riprap. Filter fabric or a filter course of gravel should be placed under the stone for larger drainage channels.

For normal channel design riprap can be sized using a method developed by the Federal Highway Administration and slightly modified for use here. Equation 303.02 gives the D_{50} size of stone (in inches) for riprap placed in a channel with average velocity "v" and depth "D".

$$D_{50}=0.0136v^3/D^{0.5}K^{1.5} \qquad \text{(Equation 303.02)}$$

K is the side slope correction factor and can be found from equation 303.03 and shall be used for all side slope placement on slopes steeper then 4H:1V. For other placement K is equal to one (1.0). θ is equal to the bank angle with the horizontal (e.g. a 1V:3H slope has a θ value of 18.43 degrees).

$$K=[1-(\sin^2\Theta/0.396)]^{0.5} \qquad \text{(Equation 303.03)}$$

Equation 303.02 is based on a safety factor of 1.2 and a stone weight of 165 lbs/ft^3. For situations other than a uniform straight channel the D_{50} size from equation 303.02 should be multiplied by a Stability Correction Factor found in the table below and used in equation 303.04.

Condition	Stability Factor
Uniform flow; straight or mildly curving reach (curve radius/channel topwidth ($R_c/T > 30$); little impact from wave action and floating debris; little uncertainty in design parameters.	1.0 - 1.2
Gradually varied flow; moderate bend curvature ($30 > R_c/T > 10$); moderate impact from waves or debris; moderate uncertainty in design parameters.	1.3 - 1.6
Approaching rapidly varied flow; sharp bend curvature ($10 > R_c/T$); significant impact from waves or debris, high flow turbulence; significant uncertainty in design parameters.	1.6 - 2.0

$$C_{SF}=(SF/1.2)^{1.5} \qquad \text{(Equation 303.04)}$$

where:

SF = stability factor
C_{SF} = stability correction factor

If the rock density is significantly different from 165 lbs/ft^3 the D_{50} size found in equation 303.02 should be multiplied by a specific gravity correction factor (C_{SG}) found in equation 303.05. S_G is the specific gravity of the stone (stone weighing 165 lbs/ft^3 has a specific gravity of about 2.65).

$$C_{SG}=[1.65/S_G-1)]^{1.5} \qquad \text{(Equation 303.05)}$$

where:

S_G = specific gravity of stone, lbs/ft^3
C_{SG} = specific gravity correction factor

The riprap layer thickness shall be a minimum of D_{100}, and the D_{85}/D_{15} value shall be less than 4.6. Stone shall be angular in shape. Riprap shall be placed so as not to be flanked by the flow. The end of the protected section should be keyed into the bank to prevent scouring failure. For riprap blanket thicknesses greater than D_{100} the following reductions in D_{50} stone size are allowed:

* for blanket thickness equal to 1.5 D_{100} the D_{50} size can be reduced 25 percent.
* for blanket thickness equal to 2.0 D_{100} the D_{50} size can be reduced 40 percent.

Channel design must account for riprap thickness in channel excavation. Channel roughness for riprap lined channels can be evaluated from (D_{50} in feet):

$$n=0.0395(D_{50})^{1/6} \qquad \text{(Equation 303.06)}$$

303.05 Design of Open Channels Using Manning's Equation

Manning's Equation may be used to size proposed open channels where backwater effects created by obstructions within the channel, or elevated tailwaters, as examples, are not of concern. Manning's Equation may be solved directly from its standard form as follows:

$$Q=\frac{1.49}{n}AR^{2/3}S^{1/2} \qquad \text{(Equation 303.07)}$$

and iterated as necessary with various values of channel geometry to obtain the desired values of flow quantity, velocity, and depth.

Engineering reference books, such as "Open-Channel Hydraulics" by V.T. Chow may be used as a guide for Manning's "n" values. Figure 303-3 provides examples of typical open channel cross-sectional configurations and contains the geometric elements of common channel sections required to determine the channel area (A), wetted perimeter (P_w), and hydraulic radius ($R = A/P_w$).

SECTION 304 CULVERTS/BRIDGES

304.01 Introduction

The design methods and criteria outlined or referred to within this section shall be used in the design and evaluation of culvert systems within the jurisdiction of this Manual. Computer models such as Federal Highway Administration's HY-8 may be used to perform culvert/bridge design computations. The design of culverts can be quite complex. Therefore only introductory material is presented herein. The designer is referred to Federal Highway Administration publication Hydraulic Design Series No.5 (HDS-5), "Hydraulic Design of Highway Culverts", Report No. FHWA-IP-85-15, for a complete treatment. Methods contained in HDS-5 shall be used for the design of culverts.

304.02 Allowable Headwater

Open culverts shall be designed to safely pass the peak discharge from the 25-year design storm event runoff from the entire contributing watershed without inundating any portion of the crossing or approach roadway. An easement must be recorded for the 25-year storm event flow areas. During the 100-year storm event, road overflow shall not exceed seven (7) inches above the centerline crown elevation of the roadway. The 100-year storm event must be checked to determine the flooded area so that a building restriction line can be shown on a record plat. The lowest elevation where water may enter any adjacent structures must be outside this delineation.

304.03 Open Culverts

Open culverts shall be sized utilizing orifice and weir flow equations where applicable for individual site conditions and storm frequencies. Inlet and outlet control nomographs for evaluation of open culvert hydraulics may also be utilized to evaluate culvert hydraulics, and have been provided in HDS-5 referred to above.

Open culverts which pose a threat of damage to property or a hindrance of public services due to backwater and/or road overflow shall be analyzed utilizing the direct-step backwater method or reservoir flood routing techniques for determination of the depth of flow over the culvert/roadway during the peak discharge from the 100-year design storm event, backwater elevations, downstream flow velocities and resulting channel scour impacts.

304.04 Culverts Operating Under Inlet Control

Inlet control for culverts may occur in two ways:

Unsubmerged: Where the headwater depth is not sufficient to submerge the top of the culvert and the culvert inlet slope is supercritical. Under these conditions, the culvert inlet acts like a weir.

Submerged: The headwater submerges the top of the culvert but the pipe does not flow full. Under these conditions the culvert inlet acts like an orifice.

In the unsubmerged inlet condition, the equation governing the culvert capacity is the weir flow equation. In the submerged inlet condition, the equation governing the culvert capacity is the orifice flow equation.

The nomographs provided by Hydraulic Design Series No. 5, Report No. FHWA- lp-85-15 may be used to determine culvert flow under inlet control conditions for common culvert materials. It should be noted by the designer that reinforced concrete pipe arch is not typically available within the Marion County or surrounding area.

304.05 Culverts Operating Under Outlet Control

Outlet control will govern in the design of open culverts when the headwater is sufficiently deep, the culvert slope sufficiently flat, and the culvert sufficiently long. There are five basic types of outlet control culvert flow conditions as depicted in Figure 304-2.

Outlet control flow conditions can be calculated based on energy balance. The Bernoulli equation may be used to solve the culvert flow problem. It can be expressed in simplified form by the following equation:

$$HW_O = T_W + H_L \qquad \text{(Equation 304.01)}$$

where:

- HW_O = Headwater depth above the outlet invert (ft)
- T_W = Tailwater depth above the outlet invert (ft)
- H_L = The sum of all the energy losses including: entrance loss, friction loss, exit loss, and losses for grates, bends, obstructions, etc.

Equation 304.01 is used to calculate the culvert capacity directly when the culvert is flowing under full flow conditions A, B or C demonstrated by Figure 304-2. Backwater calculations, beginning at the downstream tailwater elevation, may be required for conditions D or E. The downstream water surface elevation is based on the critical depth or tailwater elevation whichever is greater. Simplifications, modifications and nomographic solutions to this procedure are available in HDS-5.

304.06
Inlet/Outlet Losses

Selection of the inlet type is an important part of culvert design, particularly culverts with inlet control. Hydraulic efficiency and cost can be significantly affected by inlet conditions. The inlet coefficient K_e, is a measure of the hydraulic efficiency of the inlet, with lower values indicating greater efficiency. All methods described in this chapter, directly or indirectly, use inlet coefficients. Typical inlet coefficients are given in Table 304-1.

304.07
Outlet
Protection/Energy
Dissipation

Energy dissipators shall be employed whenever the velocity of flows leaving a storm water management facility exceeds the erosive velocity of the downstream channel system. The procedure presented in this section is taken from USDA, SCS (1975). Two sets of curves, one for minimum and one for maximum tailwater conditions, are used to determine the apron size and the median riprap diameter, D_{50}. If tailwater conditions are known, or if both minimum and maximum conditions may occur, the apron should be designed to meet criteria for both. Although the design curves are based on round pipes flowing full, they can be used for partially full pipes and box culverts. The design procedure consists of the following steps:

1. If possible, determine tailwater conditions for the channel. If tailwater is less than one-half the discharge flow depth (pipe diameter if flowing full), minimum tailwater conditions exist and the curves in Figure 304-3 apply. Otherwise, maximum tailwater conditions exist and the curves in Figure 304-4 should be used.

2. Determine the correct apron length and median riprap diameter, d_{50}, using the appropriate curves from Figure 304-3 and 304-4. If tailwater conditions are uncertain, find the values for both minimum and maximum conditions and size the apron as shown in Figure 304-5.

    a. For pipes flowing full:
    Use the depth of flow, d, which equals the pipe diameter, in feet, and design discharge, in cfs, to obtain the apron length, L_a, and median riprap diameter, d_{50} , from the appropriate curves.

    b. For pipes flowing partially full:
    Use the depth of flow, d, in feet, and velocity, v, in feet/second. On the lower portion of the appropriate figure, find the intersection of the d and v curves, then find the riprap median diameter, d_{50}, from the scale on the right. From the lower d and v intersection point, move vertically to the upper curves until intersecting the curve for the correct flow depth, d. Find the minimum apron length, L_a, from the scale on the left.

SECTION 305 STORM DRAINS/INLETS

305.01

Storm drain piping systems are networks of storm pipes, catch basins, Introduction

manholes, inlets and outfalls designed and constructed to convey surface water runoff. The hydraulic analysis of flow within storm drain piping systems typically involves analysis of flow caused by the natural forces of gravity ("gravity flow"), and hydraulic analysis of systems under pressure flow conditions.

305.02
Easements

Minimum easement widths are as provided below. More stringent requirements for stormwater easement size and additional covenants may be made by the Department based upon individual site conditions.

1. Storm Drains

Depth of Drain From Finish Grade To Crown	Diameter of Storm Drain	Minimum Easement Width
3' or less	15" or less	15'
More than 3'	15" or less	20'
3' or less	Greater than 15"	20'
More than 3'	Greater than 15"	25'

305.03
Storm Drain Pipe Design

The use of Manning's equation shall be considered acceptable for determination of storm drain pipe sizes for non-submerged conditions. The storm drain system must be capable of passing the 10-year storm event with free water surface elevations below the crown of the pipe.

Design computations of storm drain pipe systems using the Rational Formula and Manning's equation shall be submitted with the stormwater permit application on the Storm Drain Flow Tabulation Form provided by Figure 305-1 or by suitable computer program output listing giving similar information. Typical Manning's "n" values for standard storm drain materials are provided in Table 305-1. Headloss computations shall be submitted with the stormwater permit application on the form provided by Figure 305-2.

305.04
Backwater Method for Pipe System Analysis

For hydraulic analysis of existing or proposed storm drains which possess submerged outfalls, a more sophisticated design/analysis methodology than Manning's equation will be required. Various computer modeling programs are available for analysis of storm drains under these conditions. These models must be approved by the Department.

The backwater analysis method provides a more accurate estimate of pipe flow by calculating individual head losses in pipe systems which are surcharged and/or have submerged outlets. These head losses are added to a known downstream water surface elevation to give a design water surface elevation for a given flow at the desired upstream location.

Total head losses may be determined as follows:

Total head loss = frictional loss + manhole loss + velocity head loss + junction loss

Frictional loss is computed from Manning's Equation expressed in the following form:

$$S_f = \frac{(nv)^2}{2.22R^{4/3}} \qquad \text{(Equation 305.01)}$$

where:

S_f = head loss per lineal foot of drain due to friction
n = Manning's "n"
v = Flow velocity in ft/sec = Q/A
R = Hydraulic radius, ft. A/P_w

The remaining components of total head loss may be computed using standard equations, or they may be estimated using graphical solutions. Figures 305-3 and 305-4 present graphical solutions to losses in junction structures and in bends based on information provided by Baltimore County, MD. The curves are labeled "A", "B", "C" and "D". They are based on full flow in the pipes. The total loss at any junction is the sum of the applicable types of loss.

The "A" loss is the entrance or exit loss. The larger of the velocities entering or leaving the structure is used to calculate the loss. The "B" loss is not really a loss but an expedient to handle the change in velocity head during hydraulic grade line calculations. It is always the downstream velocity head minus the upstream head with the algebraic sign preserved. Where the upstream velocity head is greater than the downstream head the apparent gains may be used to offset other head losses in the structure. However, the total head loss may not be less than zero. The "C" loss represents the bend loss and is based on the higher of the two velocities. The "D" loss is the junction or combined flow losses and relates only to the incoming velocity and varies with the ratio of the Q3 and Q1 as depicted in the Figure. Intermediate values of Q3/Q1 may be interpolated.

* For cut-ins, wyes and preformed fittings use the full value of "B", "C" and "D" losses.
* For manholes and inlets use the full value of all four types of losses for pipe diameters 30" and under. For larger pipe diameters omit the "A" loss since it is accounted for in the "C" loss.
* For bends use the full value of the "B", "C" and "D" curves and increase the losses for special cases as stated on the figures.
* For junction chambers use the full values of the "B" and "C" losses; use 50% of the indicated "D" loss. Increase the "C" loss

50% for junction chambers with manholes.

305.05
Minimum Velocity

Minimum storm drain flowing velocity for full pipe flow shall be 2.5 feet-per-second (fps). The minimum slope for storm drains equal to or larger than 48 inches in diameter shall be 0.001 feet/foot.

305.06
Non-Gravity Flow Systems

Stormwater facilities shall be designed to convey stormwater runoff by gravity flow unless otherwise approved by the department.

Stormwater control systems which do not satisfy this goal would include stormwater pumping systems, and mechanical sluice gates, as examples.

Design options which do not rely upon gravity flow may be approved as a variance of Chapter 10½ of Indianapolis City Code, with documentation to the Department of the infeasibility and/or undue hardship required to install available gravity flow design options. As a minimum, the following additional information shall also be submitted with the stormwater permit application for non-gravity flow systems:

1. Identification of a lifetime maintenance schedule for the non-gravity flow system.

2. Covenants attached to the property deed which place sole responsibility for maintenance of the non-gravity flow system with the current property owner of record.

Pumping systems, where approved, shall be designed using the hydraulic methods which apply to storm drain pump systems, set forth within standard engineering texts. Non-gravity flow systems shall be designed such that should the system fail, damage to adjoining properties and facilities will be limited to the site only.

305.07
Inlet Grate Design

The design methodology used to compute the flow capacity of storm drain inlet grates shall utilize orifice and weir flow equations outlined by these Standards, with consideration given to grate open areas, and flow dimensions provided by the casting manufacturer. **The grate casting shall provide sufficient grate open area to convey the 10-Year storm event. The potential maximum depth to which stormwater may pond above the inlet grate must not threaten surrounding permanent structures and public facilities. Emergency overflow points shall be provided for inlets placed in a sumped condition.**

Roll curb and gutter inlet grates as a general rule shall be placed at a maximum interval of four-hundred (400) feet, provided a minimum 10-Year design storm flow capacity has also been

provided.

Conformance with additional requirements for design and placement of storm drain inlets within public streets and roads as set forth by the Indianapolis Department of Transportation will be required.

305.08
Gutterline Hydraulic Evaluation

Inlets in roadway gutterlines must be spaced to prevent flow from entering public road intersections. In addition, inlets should be spaced intermediately in residential street gutterlines to allow one lane (based on the lane width of the road) of traffic to remain open. Multi-lane facilities may have one travel lane on each side of the roadway flooded. The design storm for all of the conditions is the 10-year storm event. The allowable minor storm capacity of each street section may be calculated for flow in triangular gutter sections using the modified Manning's formula as follows:

$$Q = \frac{0.56}{n} S_X^{1.67} S^{0.5} T^{2.67} \qquad \text{(Equation 305.02)}$$

where:

- Q = discharge, cfs
- S_x = cross slope of the pavement (ft/ft)
- T = top width of water from vertical gutter face extending into the road, ft
- S = longitudinal grade of street (ft/ft)
- n = Manning's roughness coefficient

The above equation may also be solved utilizing the nomograph provided in Figure 305-5. Further information on other gutter configurations can be found in Federal Highway Administration, "Drainage of Highway Pavements", FHWA-TS-84-202, Hydraulic Engineering Circular No. 12.

DETENTION/RETENTION DESIGN SUMMARY REPORT

Project: ____

____

Principal Spillway Information:

Description: ____

____

Culvert

Pipe Size (in): ________ Pipe Length (ft): ________

Inlet Elevation: ________ Outlet Elevation: ________

Barrel and Riser

Barrel Size (in): ________ Barrel Length (ft): ________

Barrel Inlet Elev: ________ Barrel Outlet Elev: ________

Riser Size (in): ________ Riser Top Elev: ________

Low Flow Orifice:

Orifice Size (in): ________ Orifice Elev: ________

Weir Length (ft): ________ Weir Overtopping Elev: ________

Emergency Spillway Information:

Description: ____

____

Spillway Width (ft): ________ Spillway Elevation: ________

Protection Type: ________ Top of Embankment Elev: ________

STAGE-STORAGE-DISCHARGE RELATION			
ELEVATION (ft)	DISCHARGE (cfs)	AREA (acre)	VOLUME (acre-feet)

NARRATIVE DESCRIPTION OF DESIGN PROCESS: ____

____

____

____

____

FIGURE 302-1: Retention/Detention Design Summary Report

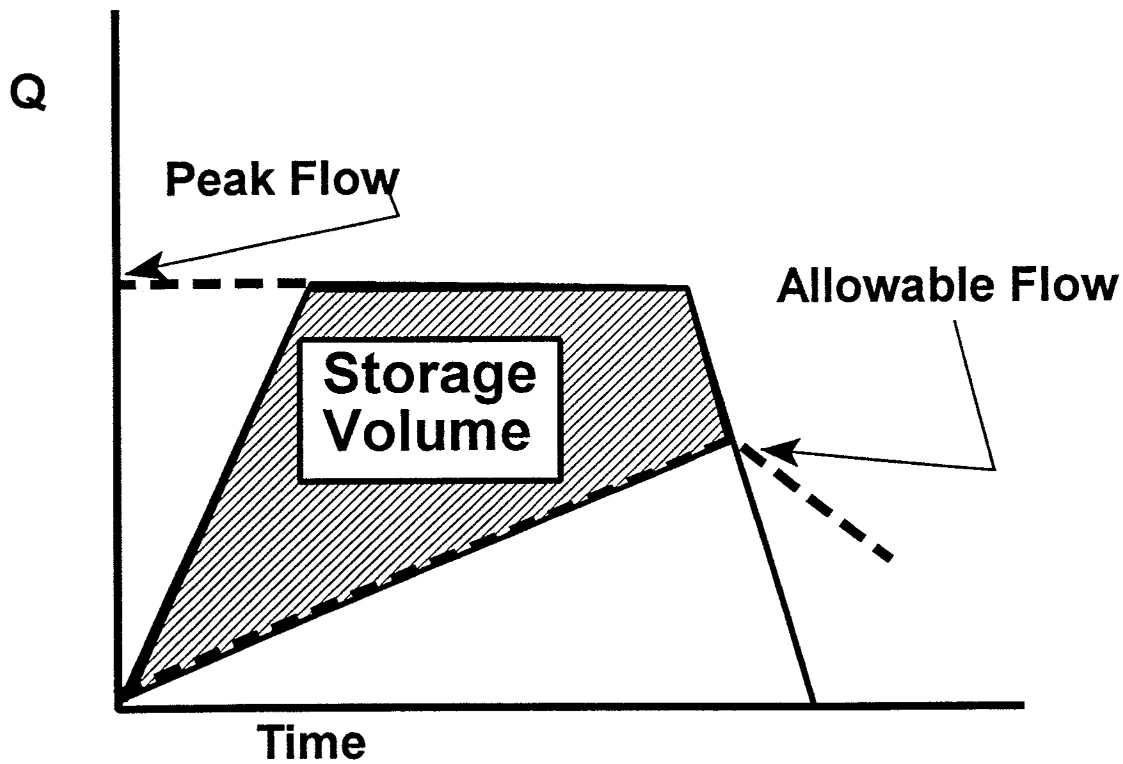


FIGURE 302-2: Modified Rational Formula Design

TABLE 303-1: Summary of Shear Stress For Various Protection Measures

Protective Cover	Underlying Soil	T (lb/ft^2)
Class A Vegetation		3.7
Class B Vegetation		2.1
Class C Vegetation		1.0
Class D Vegetation		0.60
Class E Vegetation		0.35
Bare Soil	(See Figures 303-1 and 303-2)	0.10
Hydroseeded		0.15
Woven Paper		0.45
Jute Net		0.60
Single Fiberglass		0.85
Double Fiberglass		1.45
Straw with Net		1.55
Curled Wood Mat		2.00
Plain Grass, Good Cover	Clay	N/A
Plain Grass, Average Cover	Clay	N/A
Plain Grass, Poor Cover	Clay	N/A
Grass, Reinforced with Nylon	Clay	N/A
Dycel with Grass	Clay	N/A
Petraflex with Grass	Clay	N/A
Armorflex with Grass	Clay	
Dymex with Grass	Clay	N/A
Grasscrete	Clay	N/A
Gravel		
D_{50} = 1 in.		0.40
D_{50} = 2 in.		0.80
Rock		
D_{50} = 6 in.		2.50
D_{50} = 12 in.		5.00
6 in. Gabions	Type I	
4 in. Geoweb	Type I	10
Soil Cement (8% Cement)	Type I	> 45
Dycel without Grass	Type I	> 7

TABLE 303-1: Summary of Shear Stress For Various Protection Measures

Protective Cover	Underlying Soil	T (lb/ft^2)
Petraflex without Grass	Type I	>32
Armorflex without Grass	Type I	12 - 20
Enkamat w/3 in Asphalt	Type I	13 - 16
Enkamat w/1 in Asphalt	Type I	< 5
Armorflex Glass 30 with longitudinal and lateral cables, no grass	Type I	> 34
Dycell 100, longitudinal cables, cells filled with mortar	Type I	< 12
Concrete Construction blocks, granular filter underlayer	Type I	> 20
Wedge-shaped blocks with drainage slot	Type I	> 25

Note: ft/s x 0.03048 = m/s lb/ft^2 x 47.87 = N/m^2

Source: FHWA-RD-89-110, HEC-15

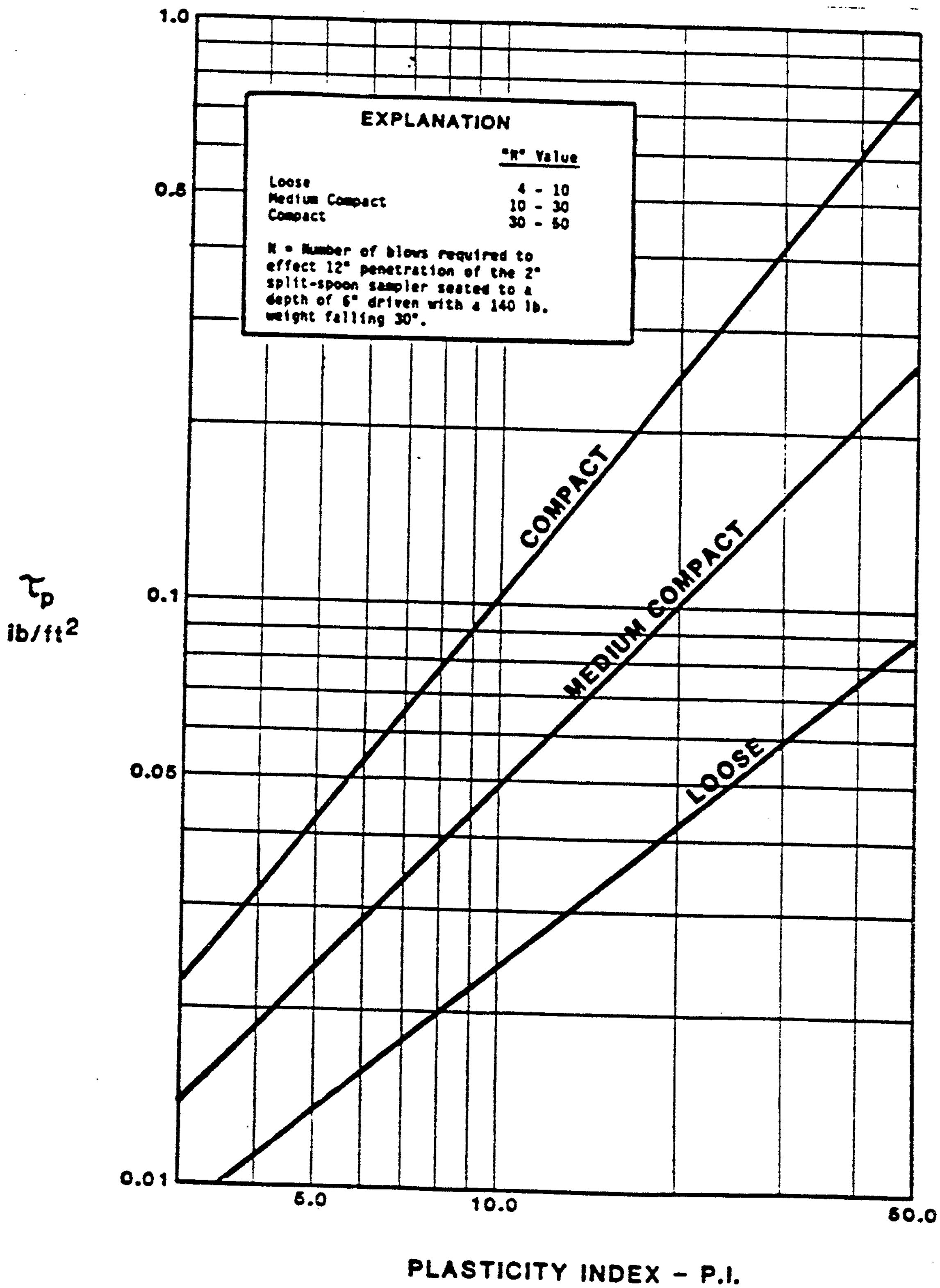


FIGURE 303-1: Permissible Shear Stress - Non-Cohesive Soils
Source: FHWA-RD-89-110, HEC-15

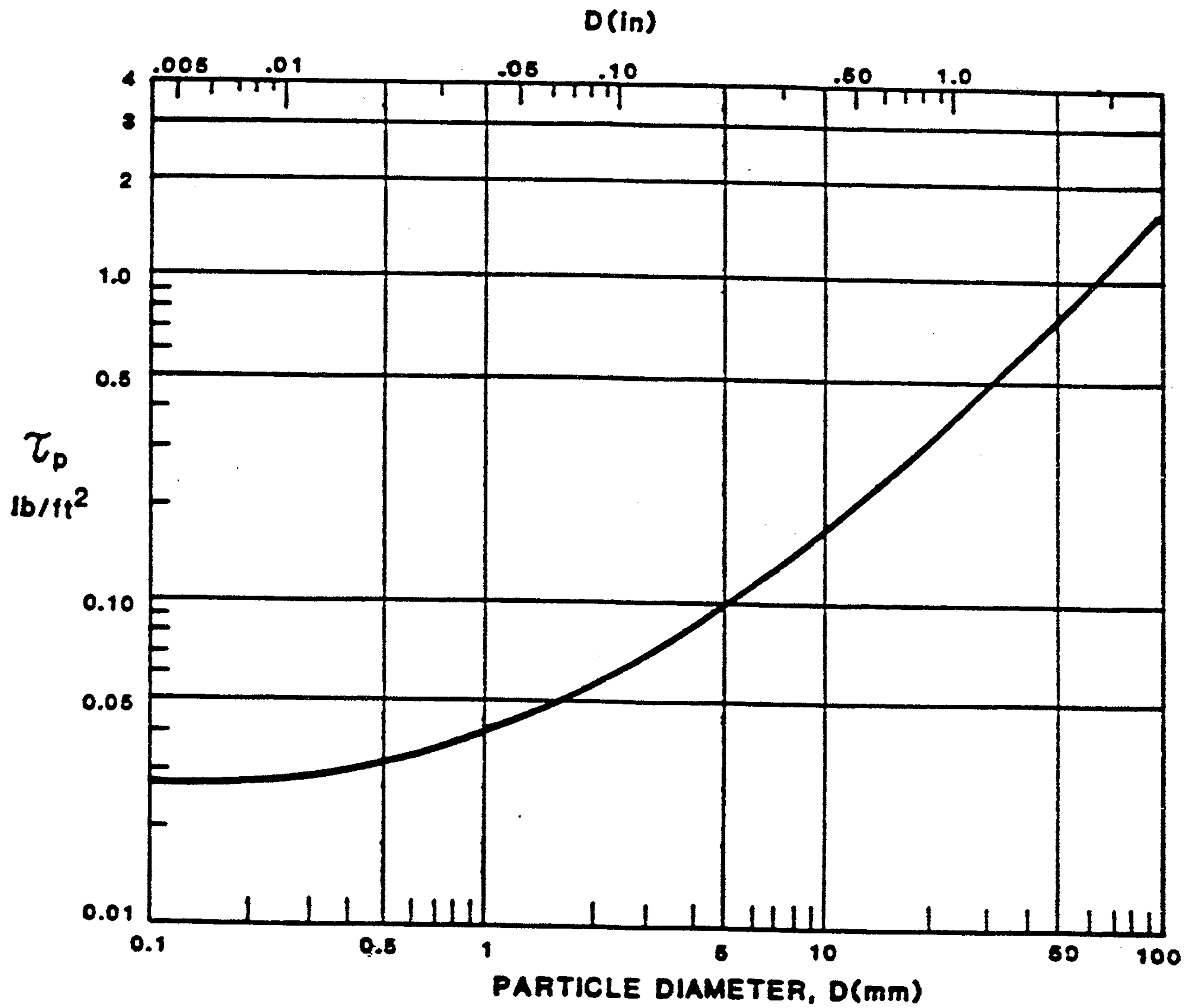
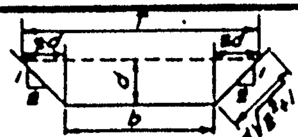
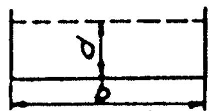
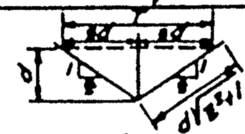
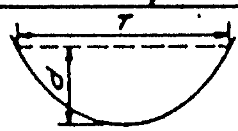
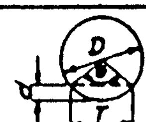
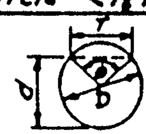


FIGURE 303-2: Permissible Shear Stress - Cohesive Soils
Source: FHWA-RD-89-110, HEC-15

Section	Area a	Wetted Perimeter P	Hydraulic Radius r	Top Width T
 Trapezoid	$bd + zd^2$	$b + 2d\sqrt{z^2+1}$	$\dfrac{bd + zd^2}{b + 2d\sqrt{z^2+1}}$	$b + 2zd$
 Rectangle	bd	$b + 2d$	$\dfrac{bd}{b + 2d}$	b
 Triangle	zd^2	$2d\sqrt{z^2+1}$	$\dfrac{zd}{2\sqrt{z^2+1}}$	$2zd$
 Parabola	$\frac{2}{3}dT$	$T + \dfrac{8d^2}{3T}$ [1]	$\dfrac{2dT^2}{3T^2 + 8d^2}$ [1]	$\dfrac{3a}{2d}$
 Circle - < ½ full [2]	$\dfrac{D^2}{8}\left(\dfrac{\pi\theta}{180} - \sin\theta\right)$	$\dfrac{\pi D\theta}{360}$	$\dfrac{45D}{\pi\theta}\left(\dfrac{\pi\theta}{180} - \sin\theta\right)$	$D\sin\dfrac{\theta}{2}$ or $2\sqrt{d(D-d)}$
 Circle - > ½ full [3]	$\dfrac{D^2}{8}\left(2\pi - \dfrac{\pi\theta}{180} + \sin\theta\right)$	$\dfrac{\pi D(360-\theta)}{360}$	$\dfrac{45D}{\pi(360-\theta)}\left(2\pi - \dfrac{\pi\theta}{180} + \sin\theta\right)$	$D\sin\dfrac{\theta}{2}$ or $2\sqrt{d(D-d)}$

[1] Satisfactory approximation for the interval $0 < \frac{d}{T} \leq 0.25$
When $d/T > 0.25$, use $p = \frac{1}{2}\sqrt{16d^2 + T^2} + \frac{T^2}{8d}\sinh^{-1}\frac{4d}{T}$

[2] $\theta = 4\sin^{-1}\sqrt{d/D}$
[3] $\theta = 4\cos^{-1}\sqrt{d/D}$
Insert θ in degrees in above equations

FIGURE 303-3: Typical Open Channel Cross-Sectional Configurations

PROJECT : ______________________

STATION : ______________

SHEET ______ OF ______

CULVERT DESIGN FORM

DESIGNER / DATE. ________ / ________

REVIEWER / DATE : ________ / ________

HYDROLOGICAL DATA

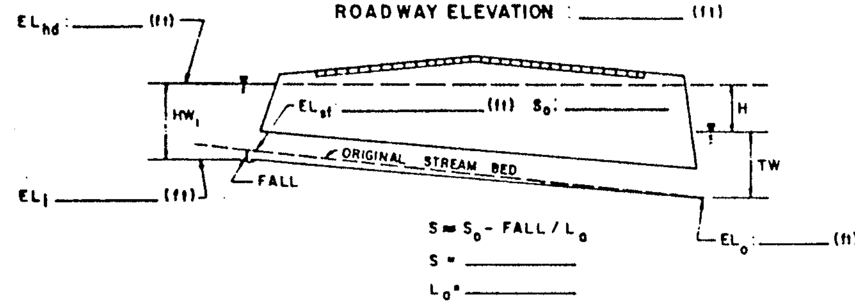
SEE ADD'L SHTS.

- ☐ METHOD : ______________
- ☐ DRAINAGE AREA. ________ ☐ STREAM SLOPE. ________
- ☐ CHANNEL SHAPE : ______________
- ☐ ROUTING ________ ☐ OTHER ________

DESIGN FLOWS/TAILWATER

R I (YEARS)	FLOW (cfs)	TW (ft)

ROADWAY ELEVATION : ________ (ft)



$S = S_o$ - FALL / L_o

S = ________

L_o = ________

CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW	FLOW PER BARREL	HEADWATER CALCULATIONS												CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
			INLET CONTROL				OUTLET CONTROL										
	Q (cfs)	Q/N (1)	HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)				

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_i/D FROM DESIGN CHARTS

(3) FALL = $HW_i - (EL_{hd} - EL_{sf})$, FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) h_o = TW or $(d_c + D/2)$ (WHICHEVER IS GREATER)

(7) $H = \left[1 + k_e + (29n^2 L)/R^{1.33}\right] v^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS :

- a. APPROXIMATE
- f CULVERT FACE
- hd DESIGN HEADWATER
- hi HEADWATER IN INLET CONTROL
- ho HEADWATER IN OUTLET CONTROL
- i INLET CONTROL SECTION
- o OUTLET
- sf STREAMBED AT CULVERT FACE
- tw TAILWATER

COMMENTS / DISCUSSION :

CULVERT BARREL SELECTED :

SIZE ______________

SHAPE ______________

MATERIAL : ________ n ________

ENTRANCE ______________

FIGURE 304-1: Culvert Rating Computation Form

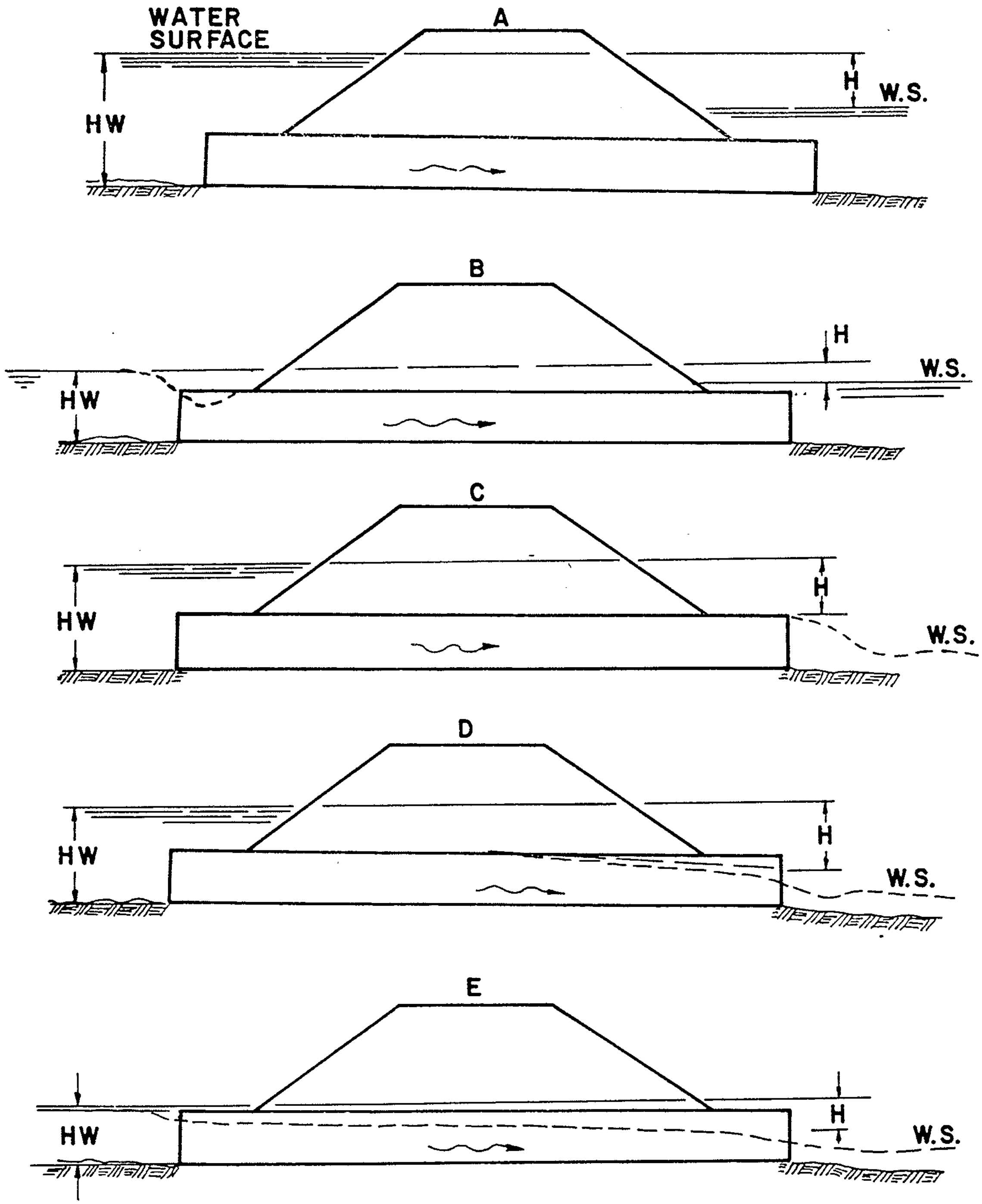


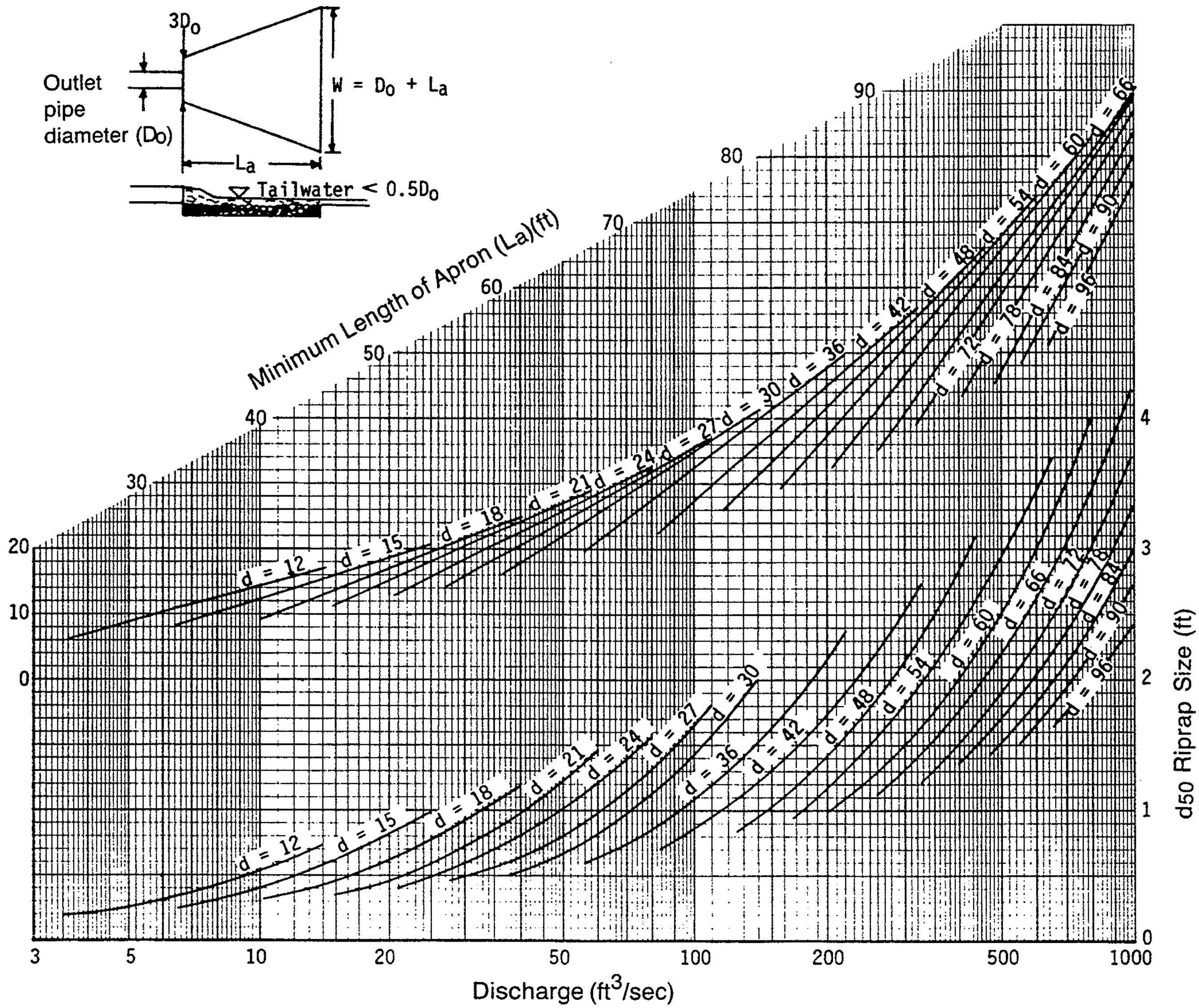
FIGURE 304-2: Culvert Flow Regimes

TABLE 304-1: Inlet Coefficients

Type of Structure and Design of Entrance	Coefficients K_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded [radius = 1/12 (D)]	0.2
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to fill slope, paved or unpaved slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of [1/12 (D)] or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of [1/12(D)] or beveled top edge	0.2
Wingwalls at 10° or 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

*Note: End Sections conforming to fill slope, made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance.

Junction Type	K_e Value
trunkline no bends at junction	0.5
trunkline with 45-degree bend	0.6
trunkline with 90-degree bend	0.8
trunkline with 1 small lateral	0.6
trunkline with 1 large lateral	0.7
2 equal entrance lines at 90-degrees	0.8
2 equal entrance lines at > 90-degrees	0.9
3 or more entrance lines	1.0

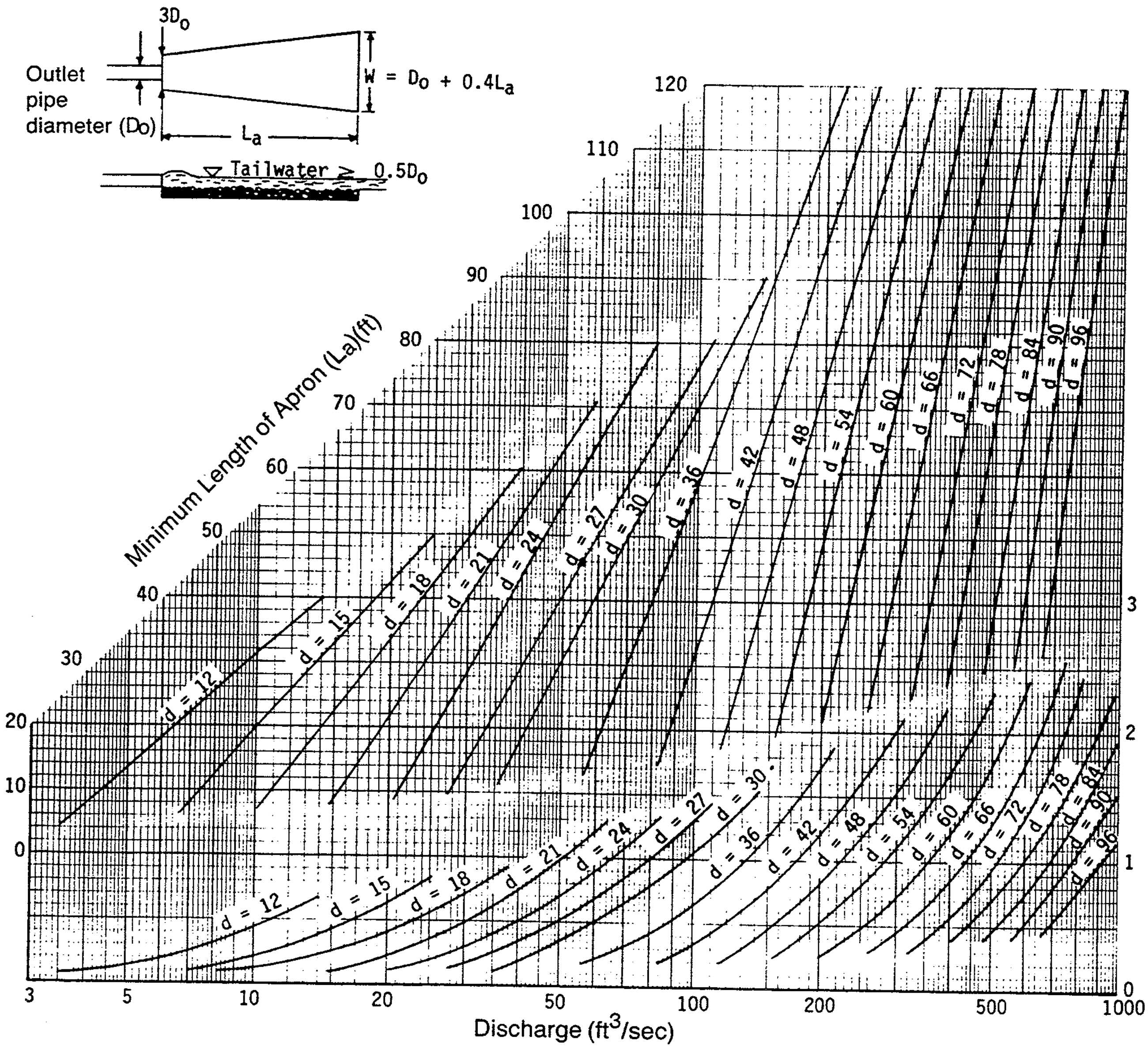


Curves may not be extrapolated.

Design of outlet protection protection from a round pipe flowing full, minimum tailwater condition (T_w < 0.5 diameter).

FIGURE 304-3: Outlet Protection with Minimum Tailwater Condition

SOURCE: North Carolina Erosion & Sediment Control Planning & Design Manual, 09/01/88



Curves may not be extrapolated.

Design of outlet protection from a round pipe flowing full, maximum tailwater condition ($T_w \geq 0.5$ diameter).

FIGURE 304-4: Outlet Protection with Maximum Tailwater Condition
SOURCE: North Carolina Erosion & Sediment Control Planning & Design Manual, 09/01/88

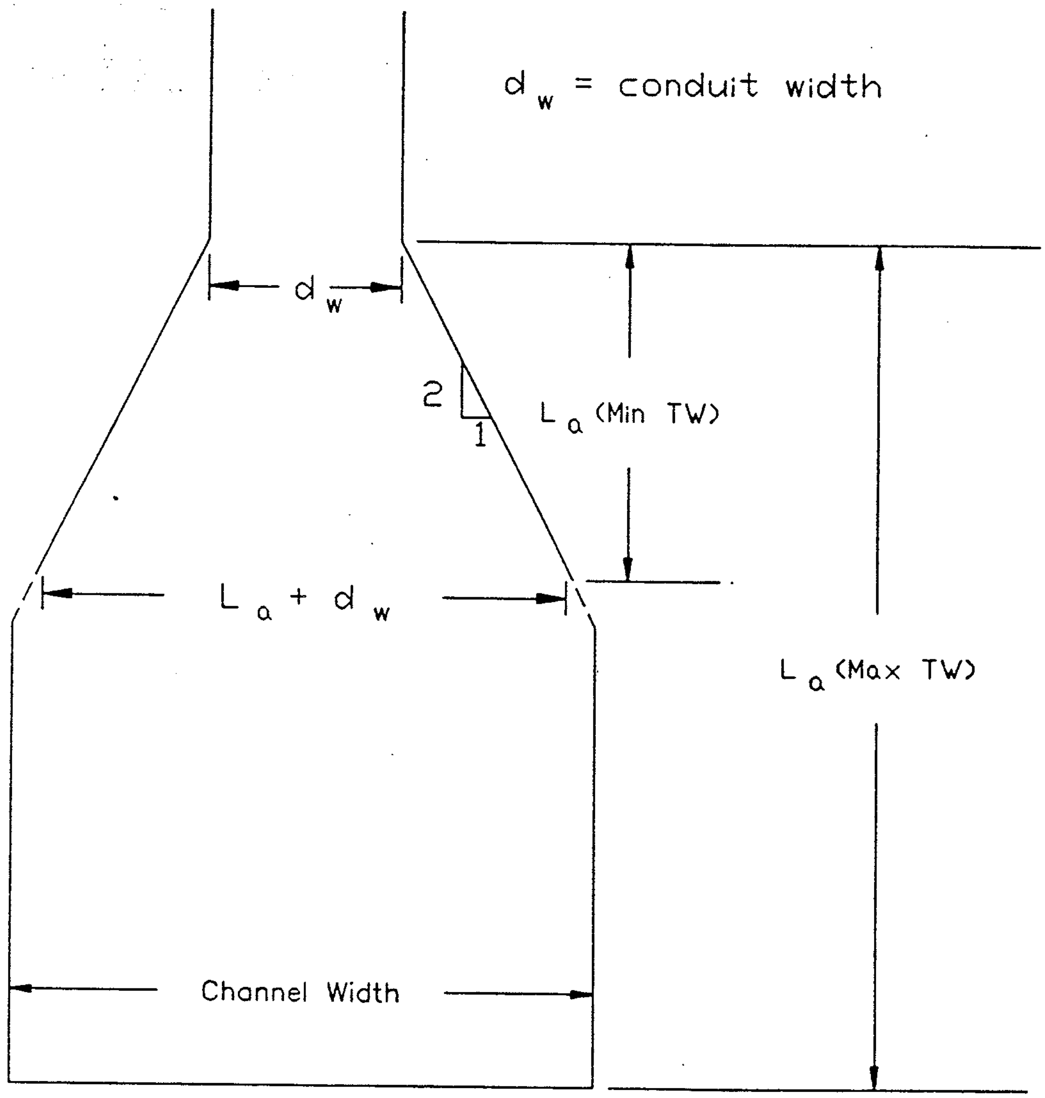


FIGURE 304-5: Riprap Apron Schematic for Uncertain Tailwater Conditions
SOURCE: Charlotte Mecklenburg Storm Water Design Manual, 07/08/93

Storm Drain Flow Tabulation Form

Structure		Length (feet)	Drainage Area "A" (acres)		Runoff Coefficient "C"	"A" x "C"		Flow Time Min		Rainfall Intensity (inches/hour)	Total Runoff (cfs)	Pipe Diameter (inches)	Full Capacity (cfs)	Velocity (fps)		Invert Elev.		Manhole Invert Drop	Slope of Sewer. (ft/ft)
From	To		Increment	Total		Increment	Total	To Upper End	In Section					Flowing Full	Design Flow	Upper End	Lower End		

FIGURE 305-1: Storm Drain Flow Tabulation Form

TABLE 305-1: Manning's "n" Values for Pipes

Pipe Material	Manning's "n"
Concrete Pipe	0.012
Concrete Boxes	0.012
Corrugated Metal Pipe or Pipe Arch	
2 2/3" x ½" Helical Corrugation	0.022
2 2/3" x ½" Annular Corrugation	
15" to 36"	0.025
42" to 96"	0.024
3" x 1" Corrugation	0.027
5" x 1" Corrugation	0.025
Structural Plate Pipe or Pipe Arch	
6" x 2" Corrugation	0.033
9" x 2 ½" Corrugation	0.035
Spiral Ribbed Corrugated Metal Pipe (7 ½" x 3/4" x 3/4")	0.013
Smooth High Density Polyethylene (HDPE) and smooth lined interior Polyvinyl Chloride (PVC)	0.012
Smooth Interior Corrugated HDPE	0.012
Ductile Iron Pipe	0.012

Headloss Computations

RUN CB TO CB	LENGTH	Q	PIPE SIZE	N	VEL	AREA	"A" MANHOLE LOSSES	"B" VELOCITY HEAD LOSSES	"C" BEND LOSSES	"D" JUNCTION LOSSES	(SXL) FRICTION LOSSES	TOTAL LOSSES	WATER SURFACE ELEVATION

FIGURE 305-2: Headloss Computations

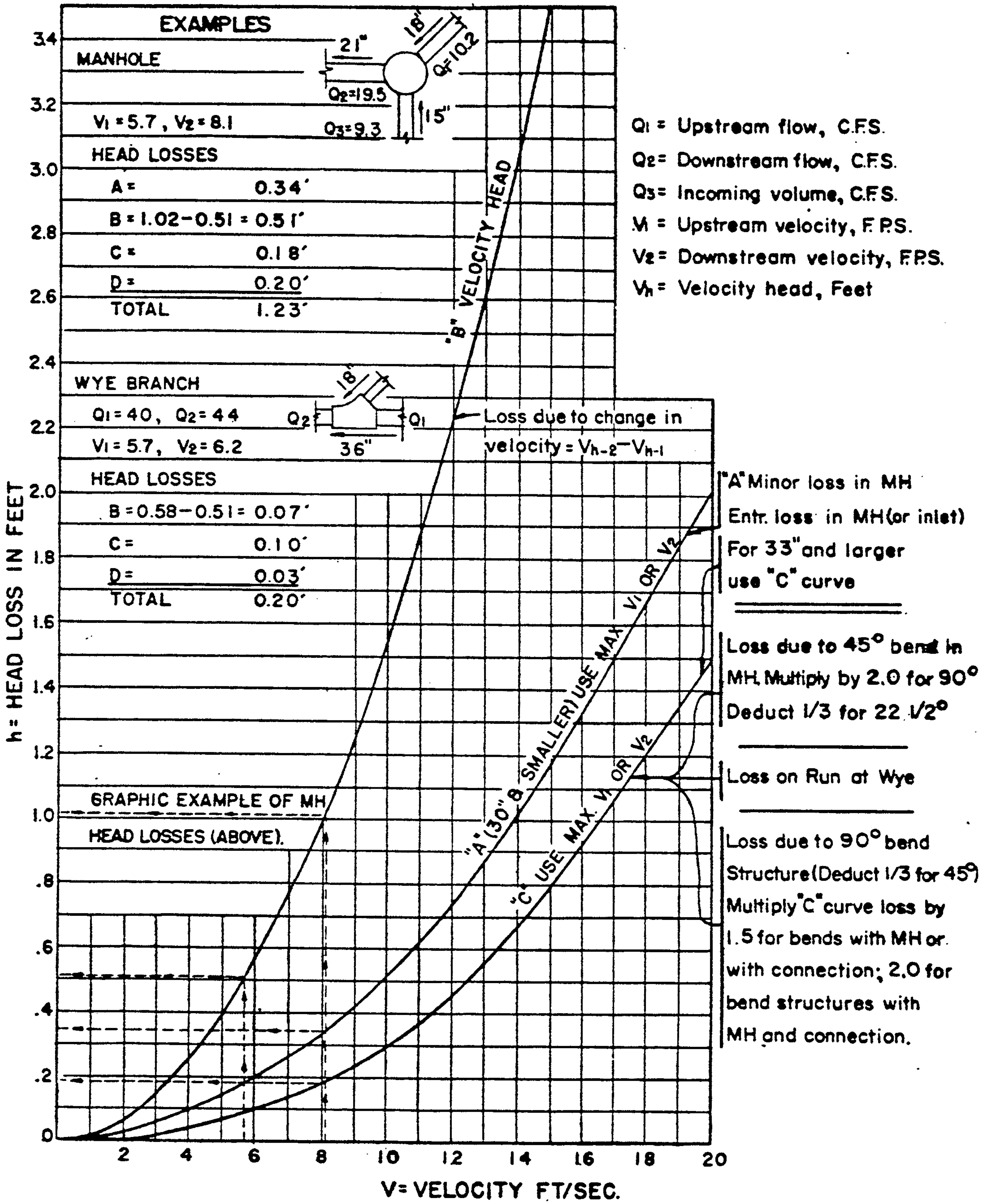


FIGURE 305-3: Manhole Loss

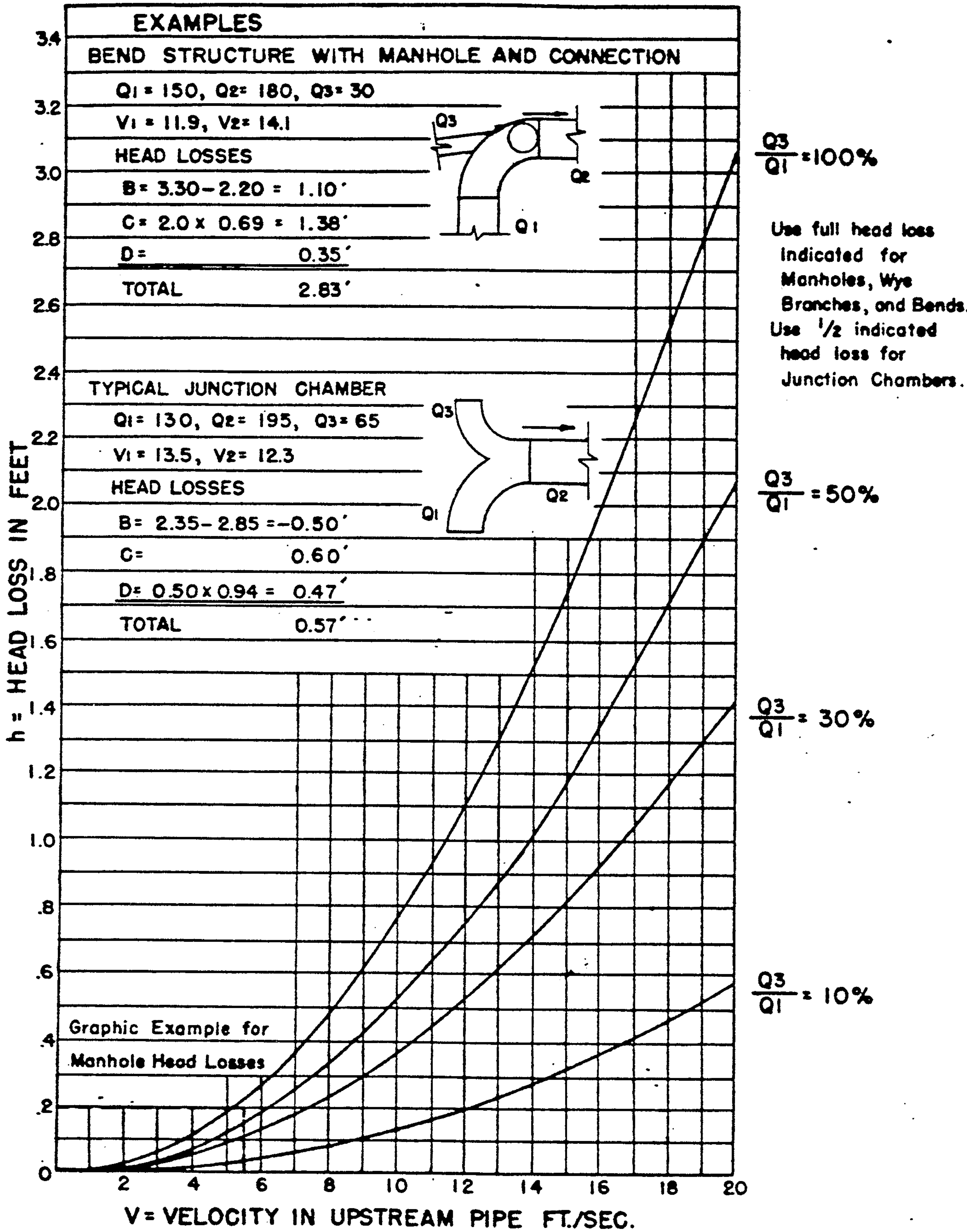


FIGURE 305-4: Junction Loss

$$Q = \frac{0.56}{n} S_x^{1.67} S^{0.5} T^{2.67}$$

EXAMPLE: GIVEN:

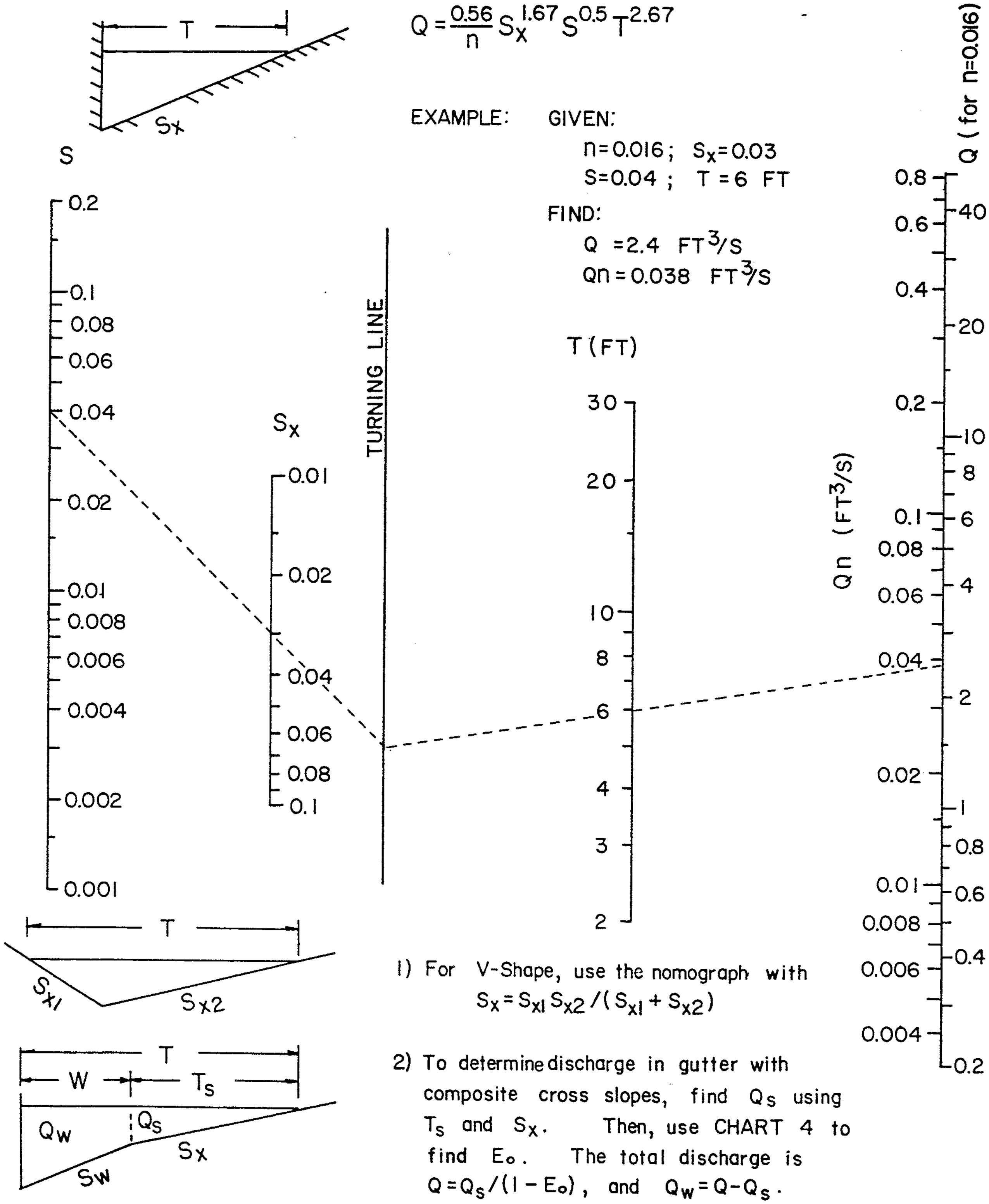
$n = 0.016$; $S_x = 0.03$

$S = 0.04$; $T = 6$ FT

FIND:

$Q = 2.4$ FT^3/S

$Qn = 0.038$ FT^3/S



1) For V-Shape, use the nomograph with $S_x = S_{x1} S_{x2} / (S_{x1} + S_{x2})$

2) To determine discharge in gutter with composite cross slopes, find Q_s using T_s and S_x. Then, use CHART 4 to find E_o. The total discharge is $Q = Q_s/(1 - E_o)$, and $Q_w = Q - Q_s$.

FIGURE 305-5: Gutterline Capacity Nomograph